FALL 2018 HOMEWORK 1 CEE 141

DUE: Monday, October 15th, 2018

NOTE: Perform calculations for LRFD method ONLY.

Chapter 1:

1-9: Provide a simple definition of structural design:

The determination of structural member sizes and connections that will result in a structure that is strong enough to resist the loads that it will experience and stiff enough to provide a serviceable structure.

1-11: Give a description of both the LRFD and ASD design approaches:

Allowable strength design (ASD) is based on the requirement that the required strength of a structure under the ASD load combinations is not to exceed the allowable strength. This allowable strength is the nominal strength of the structure divided by a factor of safety, Ω. Load and resistance factor design (LRFD) is based on the requirement that the required strength of the structure under the LRFD load combinations is not to exceed the design strength. The design strength is the nominal strength of the structure multiplied by the resistance factor, φ.

1-13: Provide an example of three serviceability limit states:

Examples of serviceability limit states include camber, deflection, drift, vibration, wind-induced motion or sway, expansion and contraction, cracking (this is for concrete not steel), connection slip, etc.

Chapter 2:

2-2: Categorize the following loads as dead load, live load, snow load, wind load, seismic load, or special load. (See Section 2.2):

a. Load on an office floor due to filing cabinets, desks, and computers. *Live*
b. Load on a roof from a permanent air handling unit. *Dead*
c. Load on stadium bleachers from students jumping up and down during a college football game. *Live*
d. Load on a building caused by an explosion. *Special – Blast*
e. Weight on a steel beam from a concrete slab that it is supporting. *Dead*
f. Load experienced by an office building in California as it shakes during an earthquake. *Seismic*
g. Load on a skyscraper in Chicago on a day with blustery conditions causing the building to sway back and forth. *Wind*

2-3: What is one source you can consult to find the snow load data for a particular region as well as maps showing wind gust data to allow you to calculate wind loads? (See Section 2.2):

ASCE 7

2-4: Where in the AISC Manual can you find a table of selected unit weights of common building materials? (See Section 2.3):

Table 17-13

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2-11: Using ASCE 7-10, determine the minimum uniformly distributed live load for a hospital operating room:

Table 4-1, 60 psf

2-13: Using ASCE 7-10, determine the minimum uniformly distributed live load for an apartment building:

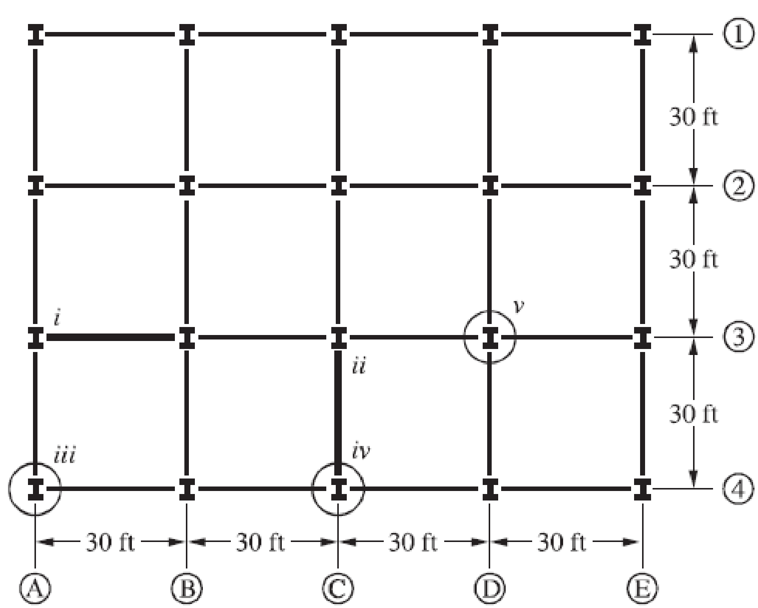
Table 4-1, 40 psf for private rooms

2-14: Determine the nominal uniformly distributed self-weight of a 6 in. thick reinforced concrete slab:

Reinforced concrete weighs 150 pounds per cubic foot. Thus (6/12)(150)=75 psf

2-15: A building has a column layout as shown in Figure P2.15 with 30ft bays in each direction. It must support dead load of 90 psf and a uniform live load of 80 psf. Determine the required strength of the members noted below for design by LRFD.

i. The beam on column line 3 between column lines A and B if the deck spans from line 2-2 to 3-3 to 4-4.
ii. The girder on column line C between column lines 3 and 4 if the deck spans from line B-B to C-C to D-D.
iii. The column at the corner on lines 4 and A.
iv. The column on the edge at the intersection of lines C and 4.
v. The interior column at the intersection of column lines D and 3.



Part a. uniform load for LRFD $w_u = 1.2(90) + 1.6(80) = 236 \text{ psf}$

	Part a. LRFD
i.	$A_I = 30(60) = 1800 \text{ ft}^2$ $L = L_o\left(0.25 + \frac{15}{\sqrt{1800}}\right) = 0.60L_o > 0.50L_0$ $w_u = 1.2(90) + 0.60(1.6)(80) = 185 \text{ psf}$ $M_u = \frac{30(0.185)30^2}{8} = 624 \text{ ft-kips}$ $V_u = 30.0(0.185)\left(\frac{30.0}{2}\right) = 83.3 \text{ kips}$
ii.	$A_I = 30(60) = 1800 \text{ ft}^2$ $L = L_o\left(0.25 + \frac{15}{\sqrt{1800}}\right) = 0.60L_o > 0.50L_0$ $w_u = 1.2(90) + 0.60(1.6)(80) = 185 \text{ psf}$ $M_u = \frac{30(0.185)30^2}{8} = 624 \text{ ft-kips}$ $V_u = 30.0(0.185)\left(\frac{30.0}{2}\right) = 83.3 \text{ kips}$
iii.	$A_I = 30(30) = 900 \text{ ft}^2$ $L = L_o\left(0.25 + \frac{15}{\sqrt{900}}\right) = 0.75L_o > 0.50L_0$ $w_u = 1.2(90) + 0.75(1.6)(80) = 204 \text{ psf}$ $P_u = 15(15)(0.204) = 45.9 \text{ kips}$
iv.	$A_I = 30(60) = 1800 \text{ ft}^2$ $L = L_o\left(0.25 + \frac{15}{\sqrt{1800}}\right) = 0.60L_o > 0.50L_0$ $w_u = 1.2(90) + 0.6\ (1.6)(80) = 185 \text{ psf}$ $P_u = 30(15)(0.185) = 83.3 \text{ kips}$

v.	$A_I = 60(60) = 3600 \text{ ft}^2$ $L = L_o\left(0.25 + \frac{15}{\sqrt{3600}}\right) = 0.50L_o > 0.50L_0$ $w_u = 1.2(90) + 0.50(1.6)(80) = 172 \text{ psf}$ $P_u = 30(30)(0.172) = 154.8 \text{ kips}$

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Chapter 3:

3-5: What happens to a steel element when it is loaded beyond the elastic limit and then unloaded? (See Section 3.3):

Plastic deformation occurs and the element is permanently deformed.

3-6: Describe the difference between the yield stress and ultimate stress of a steel element. (See Section 3.3):

The yield stress is the stress at which the steel has reached its proportional limit. Beyond this stress the element will experience plastic deformation. The ultimate stress is the peak stress that can occur in the steel. Beyond this point the element will rupture.

3-9: What are the nominal and actual depths of a W14 × 730 wide-flange member? Compare these to the nominal and actual depths of a W14 × 145. (Hint: Use your AISC Manual.):

From Manual Table 1-1, the nominal depth of the W14 x 730 is 14 inches while the actual depth is 22.4 in. From Manual Table 1-1, the nominal depth of the W14 x 145 is 14 inches while the actual depth is 14.8 in.

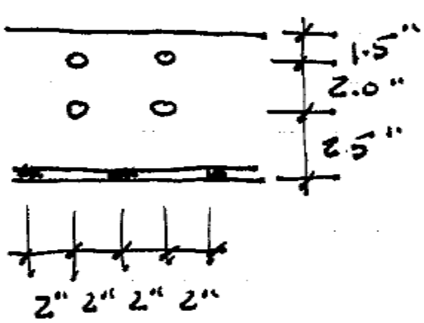
3-22: What grade of steel is most commonly used today in the production of W-shapes, and what are its yield stress and tensile stress? (see Section 3.6):

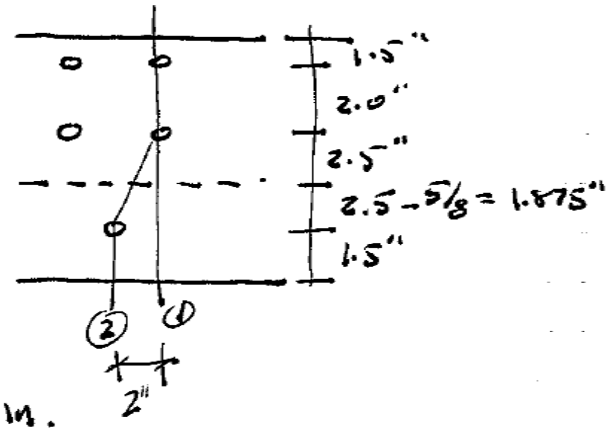
A992, 50 ksi, 65ksi

Chapter 4:

4-16:

Solution:





$$b_g = 6 + 4 - 5/8 = 9.375 \text{ in.}$$

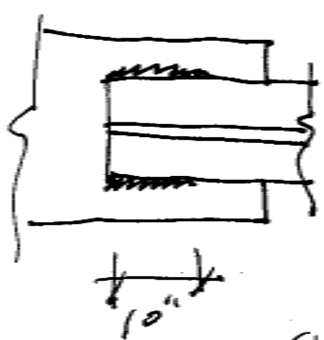
$$b_1 = 9.375 - 2\left(7/8 + 1/8\right) = 7.375 \text{ in.}$$

$$b_2 = 9.375 - 3\left(7/8 + 1/8\right) + \frac{2^2}{4(2.5 + 1.875)} = 6.604 \text{ in.}$$

$$\underline{b_n = b_2 = 6.60 \text{ in.}}$$

4-27:

Solution:



$A_g = 4.42 \text{ in}^2$

$A_n = 4.42 \text{ in}^2$

WT from W14x30 $b_f = 6.73$, $d = 13.8$

SHear LAg Factor Case 7 $b_f < 2/3\, d = 9.2$ in

$U = 0.85$

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CASE 2 $\bar{x} = 1.58$

$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.58}{10} = 0.842$

$A_e = 0.85(4.42) = 3.76 \text{ in.}^2$

yield $P_n = 50(4.42) = 221^k$

rupture $P_n = 65(3.76) = 244^k$

a) LRFD $\phi P_n = 0.9(221) = 199^k$ (yield)

$\phi P_n = 0.75(244) = 183^k$ (rupture)

$\therefore \underline{\phi P_n = 183^k}$

I. Also, answer the following problems:

1. A tension member is composed of **two** ½”x10” plates. They are connected to a gusset plate with the gusset plate between the two tension member plates, as shown in Figure 1. A36 steel and 3⁄4-inch-diameter bolts are used. Determine the nominal strength based on the net section.

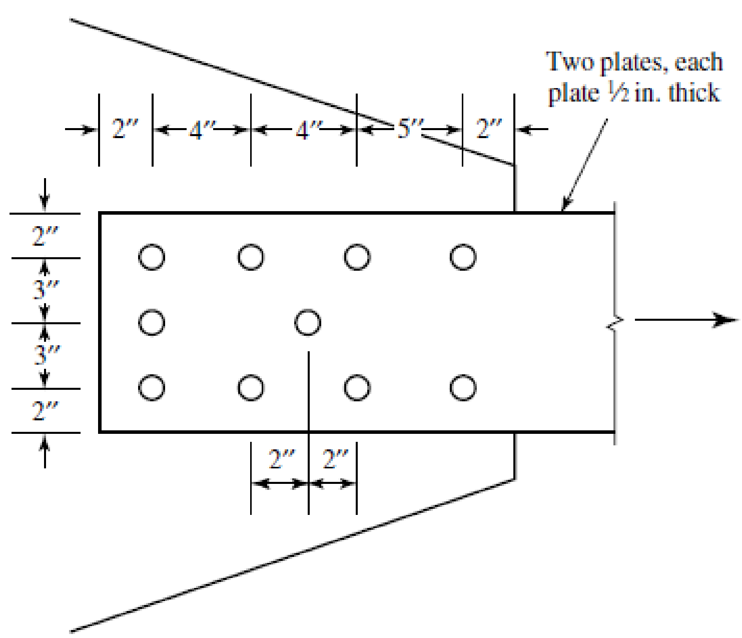

Figure 1

Solution:

Compute the strength of one plate, then double it.

Gross section: $A_g = 10(1/2) = 5.0 \text{ in.}^2$

Net section: Hole diameter $= \frac{3}{4} + \frac{1}{8} = \frac{7}{8}$ in.

Possibilities for net area:

$$A_n = A_g - \sum t \times (d \text{ or } d') = 5 - (1/2)(7/8)(2) = 4.125 \text{ in.}^2$$

or $A_n = 5 - (1/2)(7/8) - (1/2)\left[\frac{7}{8} - \frac{(5)^2}{4(6)}\right] = 4.646 \text{ in.}^2$

Because of load transfer, use $A_n = \frac{10}{9}(4.646) = 5.162 \text{ in.}^2$ for this possibility.

or $A_n = 5 - (1/2)(7/8) - (1/2)\left[\frac{7}{8} - \frac{(2)^2}{4(3)}\right] - (1/2)\left[\frac{7}{8} - \frac{(2)^2}{4(3)}\right] = 4.021 \text{ in.}^2$

Because of load transfer, use $A_n = \frac{10}{8}(4.021) = 5.026 \text{ in.}^2$ for this possibility.

The smallest value controls. Use $A_n = 4.125 \text{ in.}^2$

$$A_e = A_n U = 4.125(1.0) = 4.125 \text{ in.}^2$$

$$P_n = F_u A_e = 58(4.125) = 239.3 \text{ kips}$$

For two plates, $P_n = 2(239.3) = 478.6$ kips

The nominal strength based on the net section is $\underline{P_n = 479 \text{ kips}}$

2. Use load and resistance factor design and select a **W** shape with a nominal depth of 10" (a W10) to resist a tension dead load of 175 kips and a tension live load of 175 kips. The connection will be through the flanges with two lines of 1¼" diameter bolts in each flange, as shown in Figure 3. Each line contains more than two bolts. The length of the member is 30 feet. Use A242 steel.

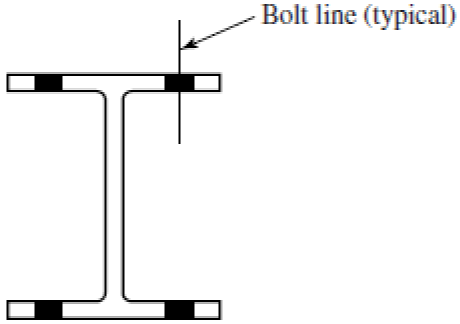

Figure 2

Solution:

From Part 1 of the Manual, all W10 shapes have a flange thickness ≤ 1.25". Therefore, from Table 2-4: Fy = 50 kasi and Fu = 70 ksi

$P_u = 1.2D + 1.6L = 1.2(175) + 1.6(175) = 490.0$ kips

Required $A_g = \dfrac{P_u}{0.9F_y} = \dfrac{490}{0.9(50)} = 10.9$ in.^{2}

Required $A_e = \dfrac{P_u}{0.75F_u} = \dfrac{490}{0.75(70)} = 9.33$ in.^{2}

Required $r_{\min} = \dfrac{L}{300} = \dfrac{30 \times 12}{300} = 1.2$ in.

Try W10 × 49

$A_g = 14.4$ in.$^2 > 10.9$ in.^{2} (OK)

$r_{\min} = r_y = 2.54$ in. > 1.2 in. (OK)

$A_n = 14.4 - 0.560(1.25 + 0.125)(4) = 11.32$ in.^{2}

$\dfrac{b_f}{d} = \dfrac{10.0}{10.0} > \dfrac{2}{3}$ $\Rightarrow$ From AISC Table D3.1, Case 7, $U = 0.90$

$A_e = A_n U = 11.32(0.90) = 10.2$ in.$^2 > 9.33$ in.^{2} (OK)

Use a W10 × 49